
Load Test and Evaluation of the Hurontario Street Bridge



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Load Test and Evaluation of the Hurontario Street Bridge

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Abstract:	This report describes the structural evaluation and load test of the Hurontario Street Bridge located in the Township of Mono near Orangeville. The structure is a one-lane bridge with a single, simply-supported span 10.87 m long. It is constructed of concrete with a slab and two parapet walls. Design drawings showing reinforcement details are not available, so an analytical evaluation of bridge strength was not possible. The bridge had been posted for a maximum weight of 16 tonnes. Load testing suggested deep beam behaviour in the parapet walls, and revealed low stress levels throughout at all load levels. Based on the maximum test load applied to the bridge, posting loads of 19, 26, and 36 tonnes at evaluation levels 1, 2, and 3, respectively, would be safe estimates of the bridge strength. However, since the measured stresses at the maximum test load were so small, it was considered safe to extrapolate the test results and predict the stress levels under full OHBD loading. These confirm that the bridge has adequate capacity to support full OHBD loading, and the removal of posting is recommended.
Comments:	
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1/ Test Summary

Bridge: Hurontario St. Bridge, Township of Mono,
MTC Site No. 4-82

Type of Bridge: Single-span, simply-supported, one-lane bridge consisting of
concrete deck and parapet walls.

Posted loaded: 16 tonnes

Span/Width: 10.87 m / 5.49 m

Angle of Skew: 0°

Date of Test: July 17, 1986

Test Engineers: Baidar Bakht, John Maheu

Job Captain: Zdislaw Knobel

Supervisor, Instrumentation: Bill Kwok

Maximum Static Test Load Applied:
One truck (Kenworth) at crawling speed.
• 2 axles: 401 kN (40.9 tonnes)
• 3 axles: 474 kN (48.3 tonnes)
• 4 axles: 546 kN (55.6 tonnes)

Responses Measured:

- Longitudinal strains in the slab and parapet walls at the critical section for flexure, and in the east parapet wall near the abutments.
- Transverse strains in the slab at one section near midspan.
- Deflections at midspan.
- Horizontal movement at the bearings.

Conclusions: The bridge can safely sustain full OHBD loading. Posting is not required.

2/ Bridge Description

2.1/ General Description

The Hurontario Street Bridge is a one-lane concrete structure crossing a small stream in the Township of Mono, northeast of Orangeville. A view of the bridge is shown in Figure 1. It consists of a concrete deck approximately 0.30 m thick and 4.57 m wide, and two concrete parapet walls 1.52 m high with varying thicknesses. Details of the cross-section are shown in Figure 2(a). The overall length of the bridge is 11.94 m with a single clear span of 10.87 m. An elevation is shown in Figure 2(b). The bridge is fixed at the north abutment and rests on steel bearing plates at the south abutment. The bearing plates, which are approximately 15 mm thick and 0.60 m wide, are located under the parapet walls.

The year of construction is not known. Since no plans are available, the amount and location of reinforcing steel are also unknown.

2.2/ Condition Survey

The bridge is generally in very good condition. There is no evidence of delamination nor is there any exposed reinforcing steel on the top surface of the exposed concrete deck, which is quite rough due to wear. There are no cracks visible on the top surface; however, there are longitudinal cracks near the parapet walls which would indicate negative transverse bending in the deck slab. Hairline cracks are likely to have been concealed by the roadway dust.

Spalling on the underside is limited to two small strips near midspan coinciding with transverse flexural cracks. Other transverse cracks were observed but no longitudinal cracking was noted on the underside.

There are longitudinal cracks on the outside faces of the parapet walls. There is a continuous crack on the east wall at approximately 0.30 m from the bottom. A similar crack on the west wall would be obscured by a utility conduit at that height. This coincides with the top of the slab, suggesting cracking along a construction joint. There are other discontinuous cracks on both the outside faces at the north end of the bridge. These are in the area between the bottom of the deck and the top of the abutment and wingwall. The bridge is integral with the abutment at this end, suggesting that the cracks may be the result of the rigidity of this support against rotation. Cracks in the north wingwalls originating near the bottom of the slab are further indicators of this rotational restraint.

The bearing plates at the south end are corroded. This would increase the frictional resistance of the bearings, and hence the restraint against horizontal movement. It is even possible that the bearings are frozen. This bearing restraint can introduce beneficial compressive stresses in the bottom of the slab which counteract the tensile strains due to positive bending [1]. Bearing restraint would not be expected to resist rotation at the south end of the bridge. The cracking in the wingwalls and lower parapet walls which was observed at the north end of the bridge, was not evident at the south end.

3/ Test Procedure

3.1/ Details of Instrumentation

The bridge was instrumented to measure longitudinal strains, transverse strains, vertical deflections and horizontal displacements at various locations. Figure 3(a) identifies the different sections where transducers were mounted. Section D is the section of maximum longitudinal live load bending moments under the critical test load. Strain transducers were mounted to measure longitudinal strains at the locations shown in Figure 3(b). Gauges on the top of the deck were limited to those positions which would not interfere with the movement of the test vehicle.

Longitudinal strains were also measured at sections A and E at the locations shown in Figure 3(c). These strains were intended to measure bearing restraint. To minimize the number of channels, only one parapet wall was instrumented, and the east wall was chosen since that is the side of the eccentric loading.

Figure 3(d) shows the locations at section B where transverse strains in the slab were measured. Section B was chosen to coincide with an axle of the test vehicle at different positions. As with section D, gauges on the top of the deck could not interfere with the movement of the test vehicle.

Section C, the bridge centreline, was instrumented for vertical deflection at the locations shown in Figure 3(e). Displacement transducers were also mounted to record horizontal movement at the bearings, shown in Figure 3(f).

All strain and displacement transducers were demountable and fastened to the bridge by means of small holes drilled into the concrete.

3.2/ Details of Loading

Details of the test vehicle and its axle weights for the three levels of loading are shown in Figure 4(a). Since the measured responses were governed by the weight of the rear dual axle, the loading was concentrated at the rear of the vehicle. The axle weights at load level 3 are the highest loads that can be carried by the rear dual axle. The total weight of the axle subgroups which were actually on the bridge for different vehicle positions are also shown in Figure 4(a).

Figures 4(b) and 4(c) show the transverse and longitudinal positions of the vehicle for the various load cases. For line 2, the eccentric loading, the east side was chosen since the deck is apparently thinner on that side.

4/ Test Results and Observed Behaviour

4.1/ Recorded Data

All the recorded strain and displacement data for the three levels of loading are given in Tables 1(a), (b) and (c).

The test was performed on a very hot and humid day with intermittent sunny periods. The strain gauges were not temperature compensating, and some of those exposed to the sunlight experienced large drifts between the beginning and end of a load sequence. Attempts to shield these gauges had limited success. One gauge on the underside of the deck, at location 15, was also defective.

The humidity also caused problems with the data acquisition system. Due to malfunction of the disk drive, it was not possible to store the data on disk or tape, nor was it possible to print the results. Since the data had to be recorded by hand, which was time-consuming, it was decided to reduce the number of load positions to two for load levels two and three.

4.2/ Longitudinal Strains

In spite of the aforementioned problems, enough reliable data was obtained on which to base an evaluation. For any load level, the largest strains recorded were the longitudinal strains at section D. The largest compressive strain in the concrete was $70 \mu\epsilon$ recorded at location 29 under load level 3, line 2 position 3. For a concrete with $f_c' = 20$ MPa and $E_c = 21,150$ MPa, this corresponds to a compressive stress of 1.5 MPa.

The largest tensile strain, $180 \mu\epsilon$, was also recorded at section D, at locations 18, 19 and 21. Tensile strains in concrete are more difficult to interpret since one or more cracks may be included in the distance covered by the strain transducer. The strength of concrete in tension is ignored, so the tensile strains are used to determine the tensile stresses in the reinforcing steel. The maximum strains and stresses in the steel will occur at a crack in concrete. The recorded strain is an average strain over the length of the transducer, which will be less than the maximum strain at a crack. However, because of the concrete cover, the depth of which is unknown, the strain at the level of the reinforcing steel will be less than that recorded at the underside of the deck. Since these two factors compensate for one another, it was assumed that the recorded tensile strain is equal to the maximum tensile strain in the reinforcing steel at the location. For $E_s = 200,000$ MPa, the maximum recorded tensile strain of $180 \mu\epsilon$ corresponds to a maximum tensile stress of 36 MPa.

The variations of the measured strains across section D, for the two transverse positions of the test vehicle and the different load levels, are shown in Figures 5(a) and (b). The curves are incomplete if the data was considered unreliable due to a large drift in the recorded strain during the load sequence and it did not seem reasonable to interpolate a value.

Figure 5(a) shows the strains under symmetric loading. The bottom tensile strains are highly unsymmetrical, with a peak under the east wheel line. This could be the result of an irregular crack distribution across the section, or an uneven distribution of the load due to the non-uniform thickness of the deck slab. The strains under eccentric loading, as shown in Figure 5(b), are unsymmetrical as expected. The strains through the depth of the parapet walls are also shown in Figure 5. The non-linear nature of the strains over the depth of the parapets suggests that the simple beam theory is not applicable to these relatively deep beams. It is clear that the assumption of plane sections remaining plane, which leads to linear strains, is not valid in this case.

The observed location of zero strain, shown in Figure 5, remained relatively constant under the various loading configurations. This would be an indication of neutral axis location provided that there were no significant axial forces in the bridge.

4.3/ Transverse Strains

The transverse strains recorded in the deck slab at section B are plotted in Figures 6(a) and (b). These transverse strains are smaller than the longitudinal strains recorded at section D under the same loading. The largest compressive strain recorded in the transverse direction was $65 \mu\epsilon$ at location 6 under load level 3, line 1, position 3. This corresponds to a maximum compressive stress of 1.4 MPa. The largest transverse tensile strain was $96 \mu\epsilon$ at location 5 under load level 3, line 2, position 3, giving a tensile stress of 19.2 MPa in the reinforcing steel. The magnitudes of these stresses indicate that transverse bending in the deck is not significant.

There was little evidence of negative bending anywhere in the deck slab. Under some loading conditions small tensile strains were recorded at both the top and bottom of the deck near one or both parapet walls. This indicates membrane action combined with little or no bending.

4.4/ Vertical Deflections

Midspan deflections under various loading conditions are shown in Figure 7. The deflections are reasonably symmetric under symmetric loading, and unsymmetrical under eccentric loading. The largest deflection recorded during testing was less than 2 mm.

4.5/ Support Restraint

The effects of support restraint were measured via the strains in the east parapet wall near both abutments and the horizontal movement near the bottom of both parapet walls at the south abutment. These values, which are listed in Table 1, were very small under all loading configurations and showed no consistency in sign or magnitude. The data cannot be considered very reliable since the measured values were small relative to the drifts in those gauges. At most, it can be concluded that restraint effects are minimal. This would include rotational restraint at the north abutment, which has probably been relieved through the observed cracking.

Without any support restraint there should be no appreciable longitudinal axial forces in the bridge. The neutral axis should therefore correspond to the observed location of zero strain shown in Figure 5.

5/ Evaluation

Since the reinforcement in this bridge is not known, the capacity cannot be determined analytically. However, it will be at least as large as the largest test load applied. Posting loads are normally based on an analysis of the bridge under evaluation loads as specified in the Ontario Highway Bridge Design Code (OHBD) [2]. The testing vehicle does not have the same configuration and axle weight ratios as the OHBD evaluation vehicles. Therefore, in order to use it as a reference vehicle for evaluating the load carrying capacity, it is necessary to establish an equivalence between the testing vehicle and the evaluation vehicles. A method, which is based on the Equivalent Base Length of the test vehicle, is described in detail in Reference 3.

5.1/ Minimum Safe Posting Loads

Since the bridge showed no signs of distress under the maximum test load, this represents the minimum load that the bridge can safely carry. Flexural effects in the bridge were governed by the rear dual axle of the test vehicle at load level 3, namely two axle loads of 200.7 kN each, 1.83 m apart. Hence it is only necessary to calculate the OHBD evaluation load which is equivalent to the rear dual axle of the test vehicle. The Equivalent Base Length, B_m , of the rear dual axle is 3.20 m, which corresponds to an equivalent OHBD evaluation truck load of 318 kN. This must be modified for a dynamic load allowance (DLA) of 0.30 for spans less than 22 m, and multiplied by a reduced live load factor of 1.25, assuming the bridge will be re-evaluated within five years. The scale down factor, F , can then be computed as the measured live load capacity of 401 kN divided by the factored OHBD evaluation load. The resulting value is 0.78, which gives posting loads of 19, 26 and 36 tonnes for levels 1, 2 and 3, respectively. The various steps in these calculations are summarized in Table 2.

5.2/ Additional Capacity

The above posting loads are minimum safe loads based on the maximum test vehicle load. Since no distress was observed at that load and the measured strains indicate low levels of stress, it is likely that the bridge has significant capacity beyond these limits. This capacity could not be tested for directly because the rear dual axle was loaded to its capacity. However, with our knowledge of material behaviour it is not unreasonable to extrapolate the test results to determine if there is sufficient capacity to support full OHBD loading. If this is the case, posting is not required. Both flexure and shear must be considered.

5.2.1/ Flexural Behaviour

Although it is not possible to calculate the ultimate flexural capacity without details of the reinforcement, it is possible to estimate the flexural stresses in the steel and concrete at a given load and compare these with the assumed material capacities. It is assumed that the material remains linearly elastic under the applied load. It is also assumed that the flexural strains have a similar transverse distribution and are in the same proportion to the theoretical moments, due to dead or live loads, as were the measured strains to the live load moments when the test vehicle was at load level 3, line 1 and position 3.

The factored dead load moment at midspan is 984 kN·m. This includes an allowance for 50 mm. of gravel which was not present during the test. The maximum factored live load moment due to the OHBDC design truck is 1190 kN·m. This includes a dynamic load allowance of 0.30 and a load factor of 1.25 based on re-evaluation within five years. The total factored design moment of 2174 kN·m is 2.4 times the maximum live load moment during the test. By multiplying the measured strains by a factor of 2.4, the strains and stresses at full factored OHBD loading can be estimated. The calculations are summarized in Table 3.

The largest compressive stress in the concrete under combined OHBDC dead and live loads is estimated at 3.6 MPa. This is 18% of the assumed compressive strength of 20 MPa. Although concrete has a quadratic stress-strain relationship, the assumption of linear behaviour is justified at these low stress levels. If a flexural performance factor, ϕ , of 0.80 is applied to the compressive strength of concrete, the predicted stress is still only 22% of capacity.

The largest tensile stress in the steel under the same design loads is estimated at 86 MPa, or 41% of the assumed yield strength of 210 MPa. The steel should be linearly elastic to stresses well beyond this. After applying the flexural performance factor for reinforced concrete, the predicted tensile stress is still only 51% of the factored yield strength. It can be concluded that the bridge has adequate flexural capacity for full OHBDC loading.

5.2.2/ Shear Capacity

The bridge was not instrumented to measure shear effects at the supports, so it is not possible to project the shear behaviour under full OHBDC loading as was done for flexure. Nor is it possible to calculate the ultimate shear capacity without knowing the reinforcement details. It is possible to calculate the shear capacity of the concrete section alone according to Section 8 of the OHBDC.

Combining the shear resistance of the concrete in the slab and parapet walls gives a minimum factored shear capacity of 1002 kN.

The factored design shear due to dead and live loads is 829 kN. Since this is larger than one third of the factored shear resistance of the concrete, a minimum shear reinforcement is required, although not in the slab portion. There is bound to be some form of shear reinforcement in the parapet walls, but the quantity is unknown. In any case, the bridge should have adequate shear capacity for full OHBDC loading.

6/ Conclusions

Based on examination of the test results and subsequent analysis it can be concluded that the Hurontario St. Bridge has sufficient capacity to support full OHBDC loading. Posting is not required. This is conditional on the bridge being re-evaluated within five years.

References

- [1] Bakht, B., and Jaeger, L.G., "Support Restraint in Slab-on-girder Bridges," to be published.
- [2] *Ontario Highway Bridge Design Code*, Ministry of Transportation and Communications (MTC), Downsview, Ontario, 1983.
- [3] Bakht, B., and Jaeger, L.G., "Behaviour and Evaluation of Pin-Connected Steel Truss Bridges," *Canadian Journal of Civil Engineering*, 14 (3), pp. 327 - 335.

Table 1 (a)/ Recorded Data

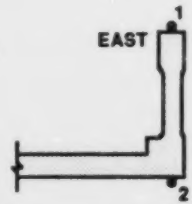
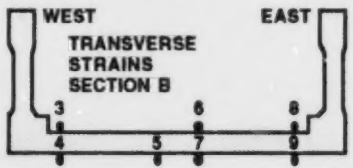
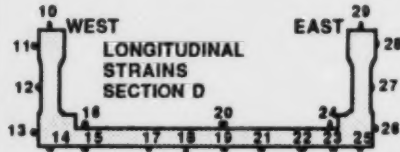
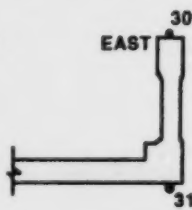
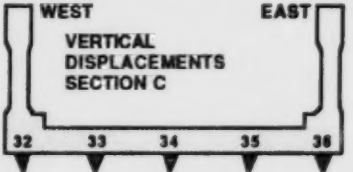
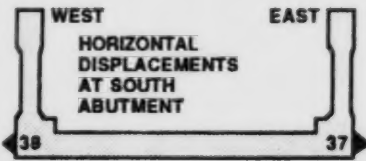
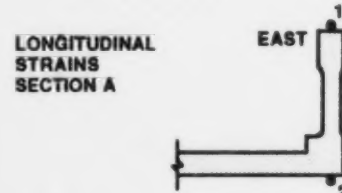
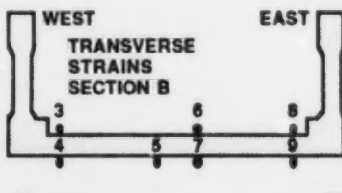
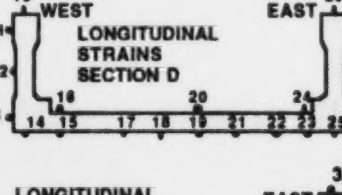
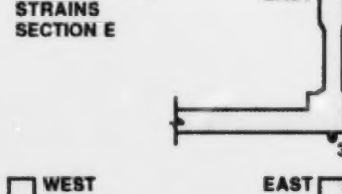
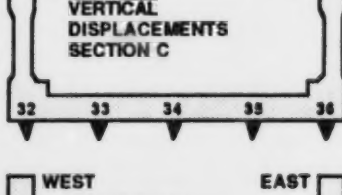
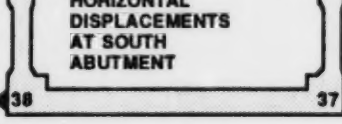
Load Level 1		Line 1				Line 2
Location	Channel Number	Position 1	Position 2	Position 3	Position 4	Position 3
STRAINS ($\mu\epsilon$)						
 LONGITUDINAL STRAINS SECTION A	1	-0.4*	-5.8*	-17.2*	-20.6*	-1.0*
	2	0.0	0.0	0.0	0.0	0.0
	3	-10.6*	-12.2*	-1.8*	1.6*	3.0
	4	7.8	6.6	4.4	4.2	0.5*
	5	26.8	25.6	34.4	35.2	46.5
	6	-43.6*	-47.2*	-46.8*	-36.4*	-27.0*
 TRANSVERSE STRAINS SECTION B	7	12.4*	12.8*	18.2*	18.6*	16.5
	8	-18.8*	-18.6*	-8.4*	-0.2*	-7.0*
	9	13.8	13.6	7.4	8.2	23.5
	10	-22.0*	-15.0*	-29.0*	-7.0*	-16.5
	11	-13.4	-17.8	-18.2	-11.6	-7.5*
	12	-8.6	-11.2	-11.8	-2.4	-5.0*
 LONGITUDINAL STRAINS SECTION D	13	4.6	6.2	6.8	6.4	5.5*
	14	9.6	12.2	11.8	7.4	7.5
	15	-4.0*	1.0*	4.0*	3.0*	-23.5*
	16	-1.4*	-3.8*	-3.2*	-0.6*	0.0
	17	5.4	48.8	50.2	6.6	34.5
	18	3.8	48.6	50.4	7.2	58.5
	19	6.4	63.8	64.2	8.6	60.0
	20	28.2*	3.4*	11.6*	34.8*	-25.5
	21	12.4	80.8	81.2	23.6	67.0
	22	2.8	12.6	13.4	4.2	30.5
 LONGITUDINAL STRAINS SECTION E	23	5.6	11.2	11.8	12.4	18.0
	24	-5.0*	-3.0*	-4.0*	-1.0*	-5.5*
	25	4.6	6.2	6.8	4.4	7.5
	26	5.6	6.2	4.8	2.4	9.5
	27	-5.4	-8.8	-8.2	-4.6	-8.0
	28	-19.4*	-24.8*	-23.2*	-15.6*	-25.0
 VERTICAL DISPLACEMENTS SECTION C	29	-32.2	-38.4	-38.6	-18.8	-26.5
	30	-6.6*	-2.2*	-2.8*	-0.4*	-2.0*
	31	2.6*	2.2*	0.8*	-0.6*	1.0*
	DISPLACEMENTS (mm)					
 HORIZONTAL DISPLACEMENTS AT SOUTH ABUTMENT	32	-0.42	-0.31	-0.33	-0.29	-0.28
	33	-0.62	-0.68	-0.69	-0.52	-0.62
	34	-0.77	-0.87	-0.88	-0.63	-0.85
	35	-0.61	-0.62	-0.65	-0.50	-0.77
	36	-0.49	-0.47	-0.40	-0.27	-0.71
	37	-0.04*	-0.07*	-0.01*	0.06*	-
38	-0.05*	-0.03*	0.03*	0.05*	0.04*	

Table 1 (b)/ Recorded Data

Load Level 2		Line 1		Line 2	
Location	Channel Number	Position 2	Position 3	Position 2	Position 3
STRAINS ($\mu\epsilon$)					
 LONGITUDINAL STRAINS SECTION A	1	-3.0*	-11.0*	-1.0*	-14.0*
	2	0.0	0.0	0.0	0.0
	3	-7.0	-8.0	-2.0	5.0
	4	9.7	4.3	5.0*	0.0*
	5	39.7	59.3	35.3	71.7
	6	-21.3*	-8.7*	-52.3	-60.7*
 TRANSVERSE STRAINS SECTION B	7	20.7	29.3	20.0	29.0
	8	-24.3*	-4.7*	-36.7*	-15.3*
	9	19.0	13.0	25.7	45.3
	10	-39.0	-37.0	-22.7	-26.3
	11	-24.7*	-24.3*	-18.0	-20.0
	12	-19.0	-16.0	-11.7	-13.3
 LONGITUDINAL STRAINS SECTION D	13	7.3*	8.7*	4.3*	3.7*
	14	22.7	20.3	14.0	13.0
	15	8.3*	8.7*	-.*	-.*
	16	0.0	-1.0	0.0	1.0
	17	86.0	82.0	58.3	59.7
	18	92.0	91.0	102.3	106.7
 LONGITUDINAL STRAINS SECTION E	19	114.0	110.0	112.0	113.0
	20	-49.7*	-42.3*	-56.0*	-53.0*
	21	142.7	142.3	113.0	115.0
	22	33.7	34.3	59.7	61.3
	23	18.3	19.7	31.3	32.7
	24	-4.7	-4.3	-9.3*	-11.7*
 VERTICAL DISPLACEMENTS SECTION C	25	9.7	10.3	14.7	15.3
	26	9.7	9.3	17.7	15.3
	27	-14.7	-13.3	-12.0	-13.0
	28	-40.0	-42.0	-43.7	-46.3
	29	-47.7*	-40.3*	-52.7	-57.3
	30	-9.7	-3.3	-4.3	-2.7
 HORIZONTAL DISPLACEMENTS AT SOUTH ABUTMENT	31	2.0	0.0	3.3*	2.7*
	DISPLACEMENTS (mm)				
	32	-0.57	-0.58	-0.47	-0.41
	33	-1.08	-1.13	-0.91	-0.91
	34	-1.35	-1.41	-1.32	-1.37
	35	-1.18	-1.21	-1.26	-1.30
36	-0.66	-0.64	-0.91	-0.86	
37	0.03*	0.02*	-0.08*	-0.07*	
38	0.02*	0.03*	0.00	0.00	

* Channels recording large drifts during load sequence.

Table 1 (c)/ Recorded Data

Load Level 3		Line 1		Line 2	
Location	Channel Number	Position 2	Position 3	Position 2	Position 3
LONGITUDINAL STRAINS SECTION A	1	1.0*	-4.0*	-0.3*	0.3*
	2	0.0	0.0	0.0	0.0
	3	-9.0*	4.0*	-4.0	5.0
	4	16.3	11.7	8.3	-0.3
	5	55.7	79.3	45.3	95.7
	6	-33.0	-65.0	-29.3	-50.7
	7	31.7	59.3	26.0	48.0
	8	-13.3*	13.3*	-27.3	-4.7
	9	25.7	16.3	33.3	67.7
	10	-51.3	-53.7	-36.0	-33.0
TRANSVERSE STRAINS SECTION B	11	-35.7	-38.3	-24.7	-25.3
	12	-20.0	-19.0	-17.0*	-14.0*
	13	15.7	15.3	7.3*	9.7*
	14	37.3	31.7	20.0	18.0
	15	21.3*	7.7*	5.3*	6.7*
	16	39.0*	-35.0*	0.7*	0.3*
	17	124.7	110.3	85.0	89.0
	18	141.3	136.7	180.3	166.7
	19	168.0	157.0	171.0	179.0
	20	-65.3	-52.7	-59.3	-56.7
LONGITUDINAL STRAINS SECTION D	21	173.0	165.0	166.0	180.0
	22	52.7	50.3	76.0	91.0
	23	27.3	27.7	43.7	49.3
	24	0.0	0.0	-0.3	-1.7
	25	13.0	12.0	20.0	19.0
	26	16.0	11.0	23.3	21.7
	27	-18.0	-17.0	-20.3	-19.7
	28	-56.0	-56.0	-66.7	-69.3
	29	-52.0	-58.0	-67.7	-70.3
	30	-6.0	-3.0	-6.0	-4.0
LONGITUDINAL STRAINS SECTION E	31	4.0	2.0	4.0	1.0
	32	DISPLACEMENTS (mm)			
	33	-0.81	-0.85	-0.61	-0.66
	34	-1.51	-1.54	-1.19	-1.40
	35	-1.87	-1.94	-1.69	-1.86
	36	-1.61	-1.62	-1.67	-1.84
	37	-0.89	-0.86	-1.22	-1.22
	38	0.07*	0.06*	-0.04*	-0.03*
	39	-0.03*	0.03*	0.01	-0.03*
	40				
VERTICAL DISPLACEMENTS SECTION C	32				
	33				
	34				
	35				
	36				
	37				
	38				
	39				
	40				
	41				
HORIZONTAL DISPLACEMENTS AT SOUTH ABUTMENT	32				
	33				
	34				
	35				
	36				
	37				
	38				
	39				
	40				
	41				

* Channels recording large drifts during load sequence.

Table 2/ Calculation of Minimum Safe Posting Loads



Dual axle that can be safely sustained by the bridge.

Equivalent Base Length Bm (m)		3.20
Total Weight $\bar{W}$ (kN)		401
DLA		0.30
Equivalent Static Weight $\bar{W}_s = \frac{\bar{W}}{1 + \text{DLA}}$ (kN)		308
Equivalent OHBDC Truck Load, W (kN)		318
$F = \frac{\bar{W}_s}{1.25 W}$		0.78
Posting Loads (t)	Level 1 Level 2 Level 3	19 26 36

**Table 3/ Estimated Flexural Strains
and Stresses under OHBD Loading**

		CONCRETE (COMPRESSION)	STEEL (TENSION)	
E (MPa)		21 150	200 000	
f _c (MPa)	F _y (MPa)	20	210	
LOAD TEST RESULTS	Test Location	29	21	
	Maximum Live Load Strain (μ ε)	70	180	
	Maximum Live Load Stress (MPa)	1.5	36.0	
ESTIMATED OHBD LOADING	Factored Dead Plus Live Load Strain (μ ε)	168	432	
	Factored Dead Plus Live Load Stress (MPa)	3.6	86.4	
	$\frac{\sigma_c}{f_c}$	$\frac{\sigma_s}{F_y}$	0.18	0.41
	$\frac{\sigma_c}{\phi f_c}$	$\frac{\sigma_s}{\phi F_y}$	0.22	0.51

$\phi = 0.80$ for reinforced concrete

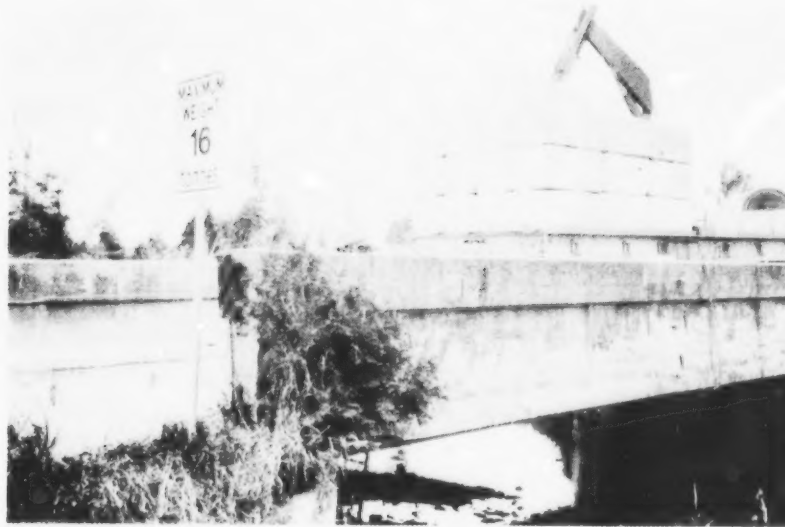


Figure 1/ View of the Hurontario St. Bridge under Maximum Test Load

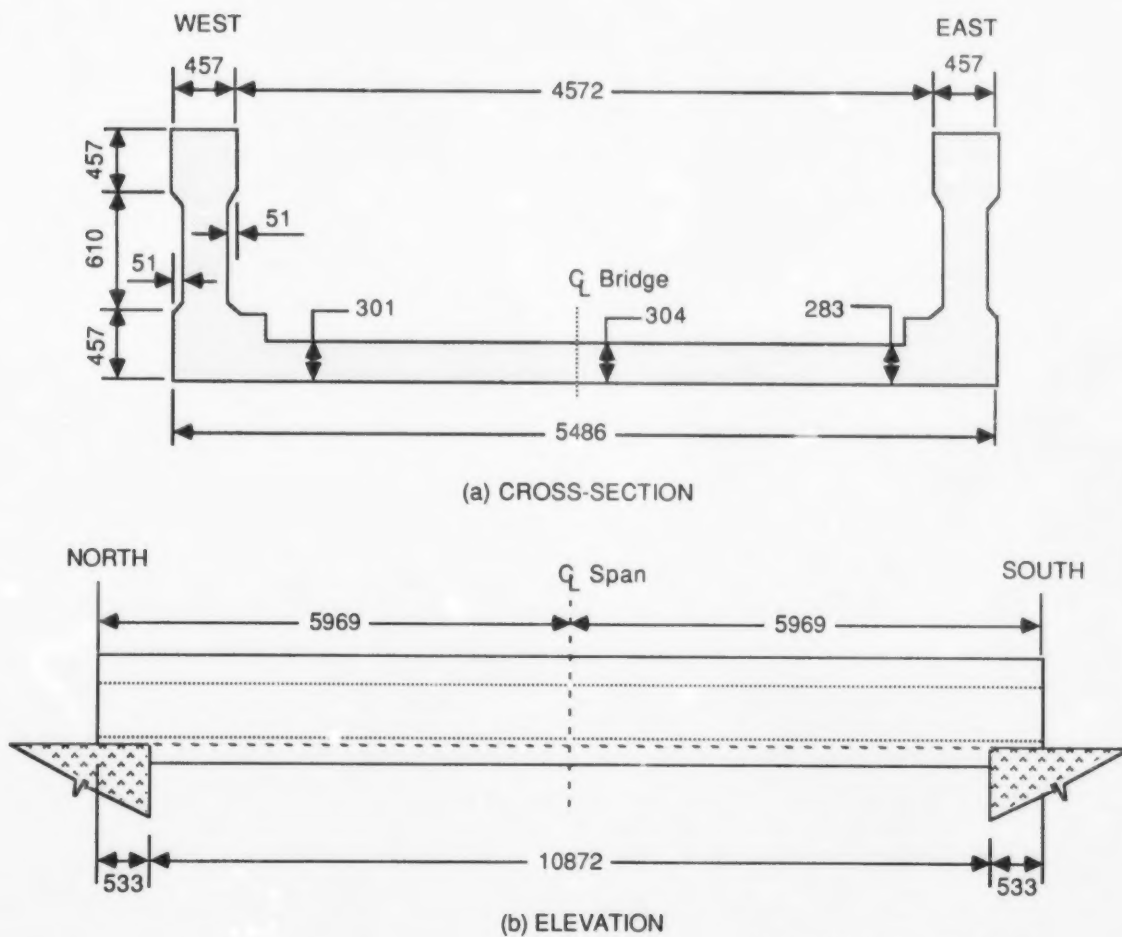


Figure 2/ Bridge Details

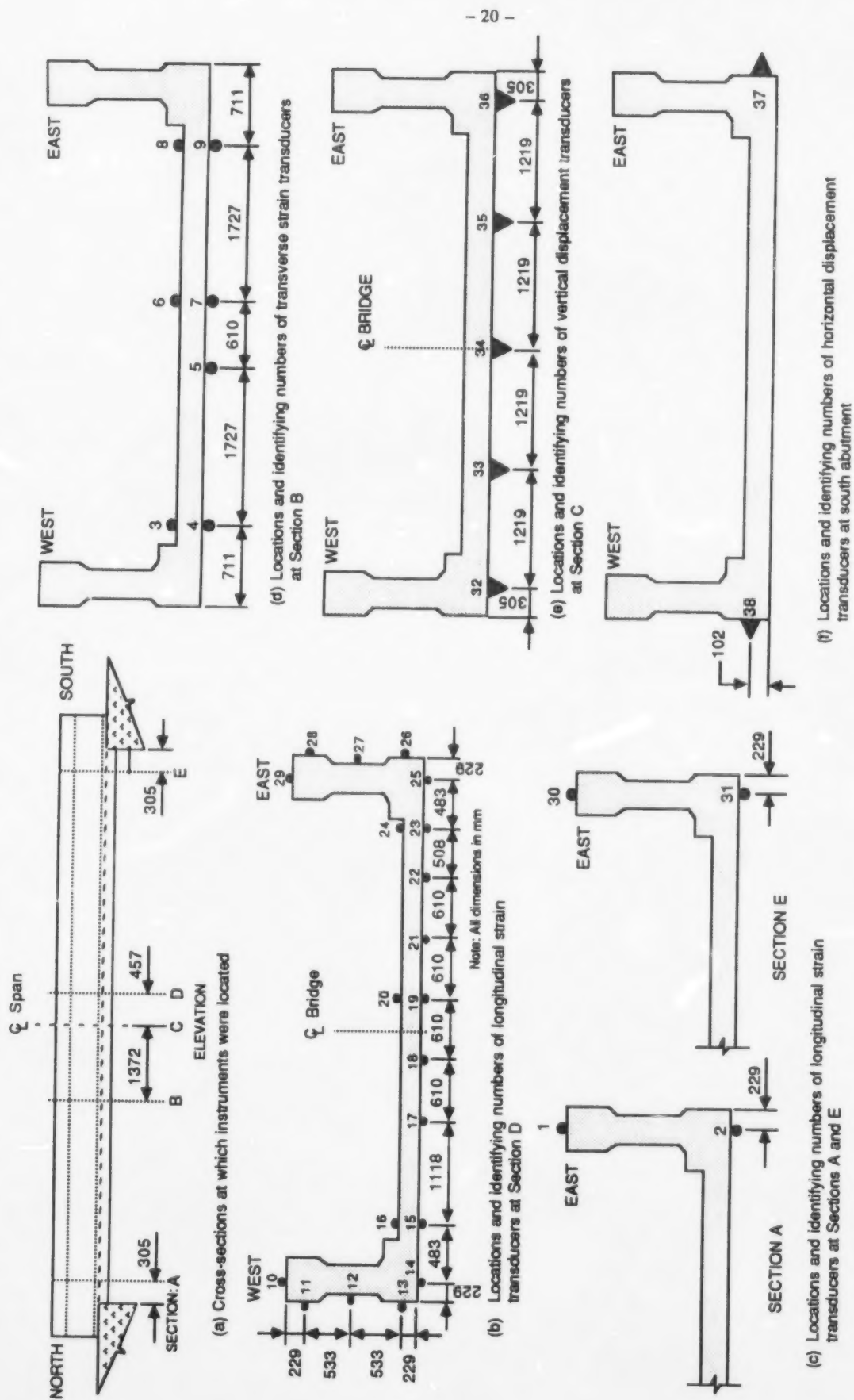
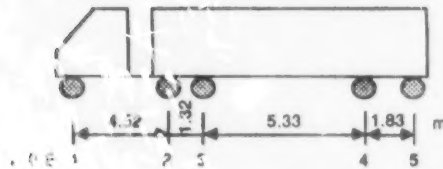
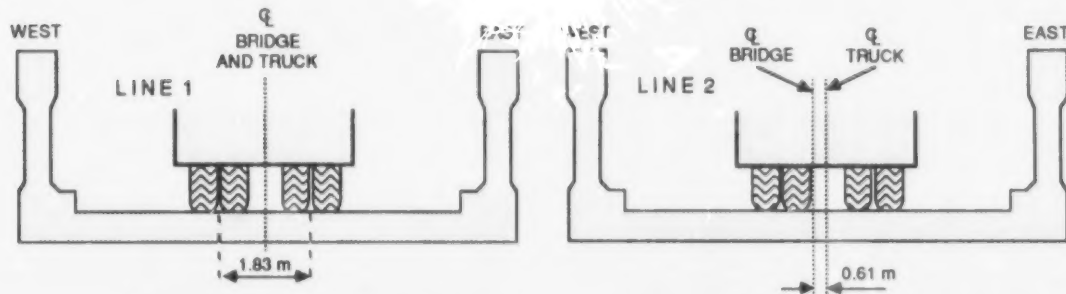


Figure 3/ Instrumentation Details

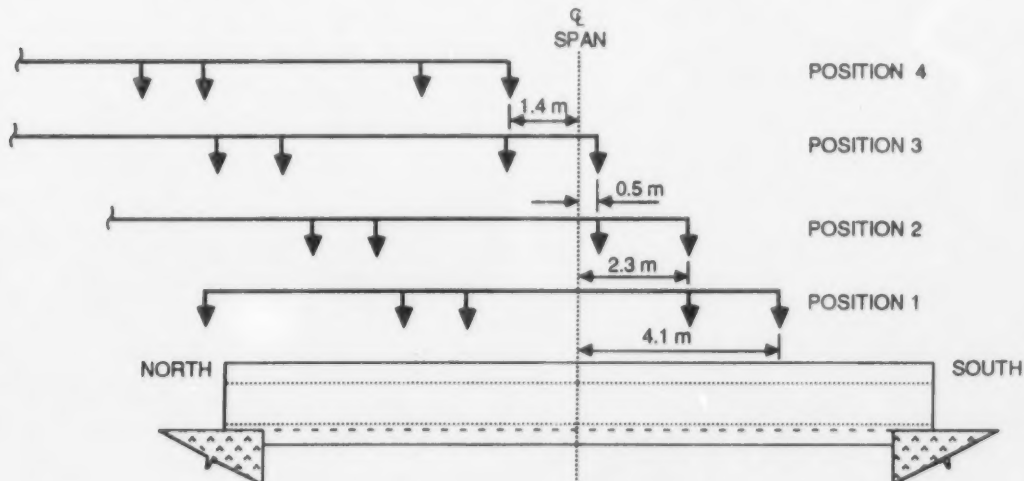


LOAD POSITION	AXLE 1	AXLE 2	AXLE 3	AXLE 4	AXLE 5	TOTAL WEIGHT	SUB-GROUP WEIGHT
1	45.3	80.8	48.1	94.3	94.3	368.0	
					94.3		322.2
					94.3		255.4
					94.3		188.6
2	45.3	80.8	111.7	147.7	147.7	480.0	
					147.7		434.2
					147.7		364.8
					147.7		295.4
3	46.3	72.1	200.7	200.7	200.7	591.9	
					200.7		545.6
					200.7		473.5
					200.7		401.4

(a) Test vehicle configuration and axle weights

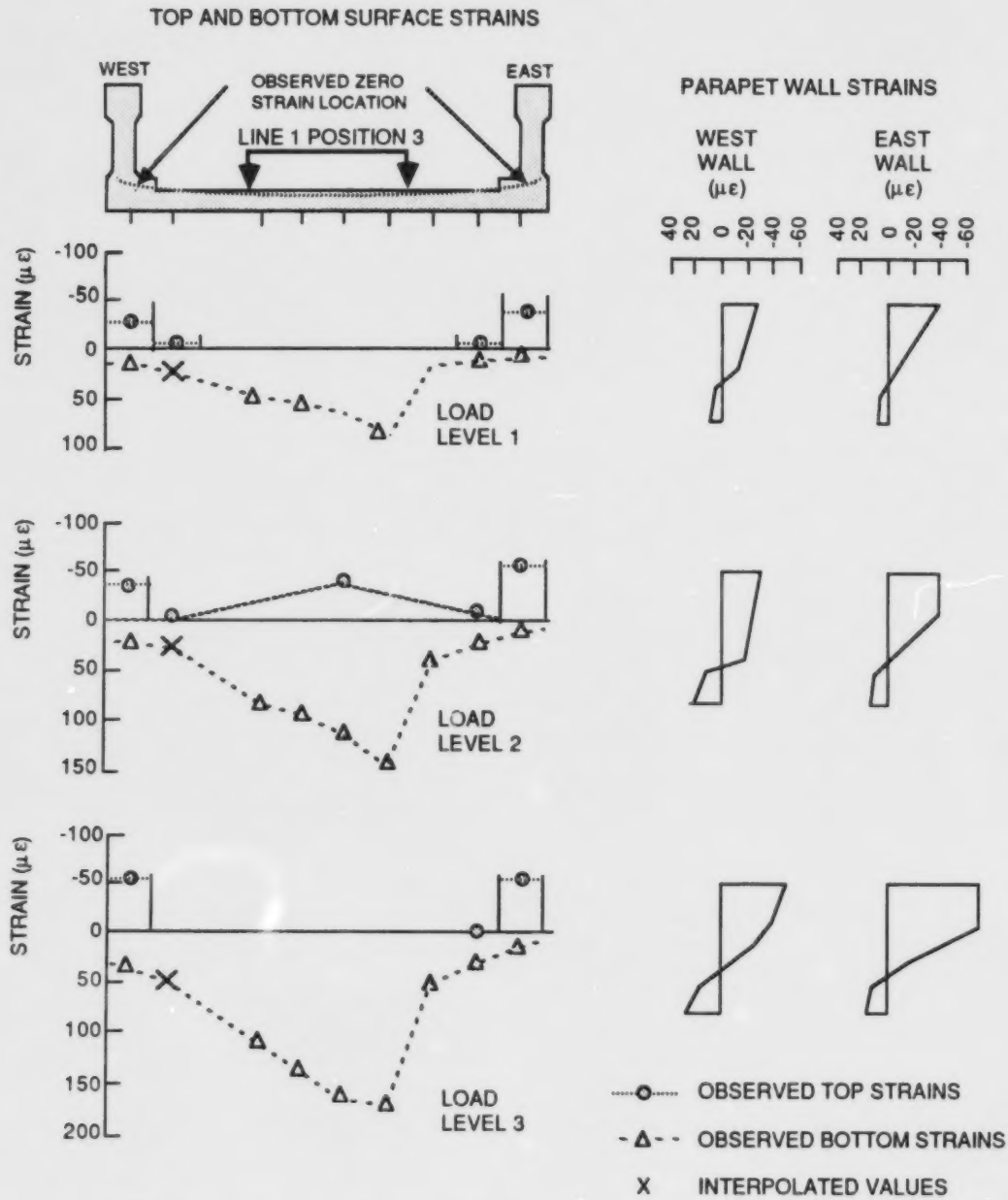


(b) Transverse positions of test vehicle



(c) Longitudinal positions of test vehicle

Figure 4/ Loading Details



NOTE: Compressive strains are shown as -ve

Figure 5(a)/ Longitudinal Strains at Section D, Symmetric Loading

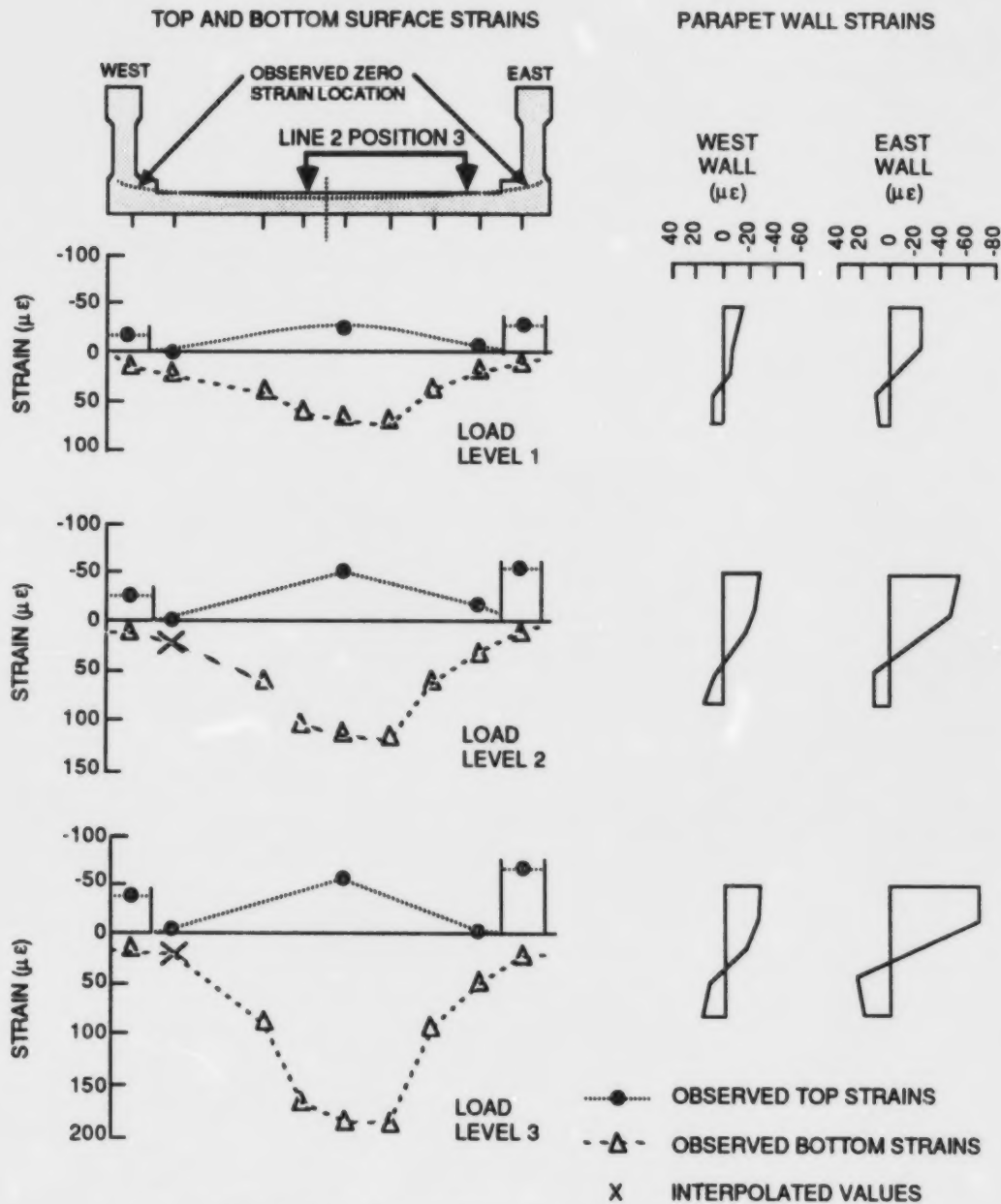


Figure 5(b)/ Longitudinal Strains at Section D, Eccentric Loading

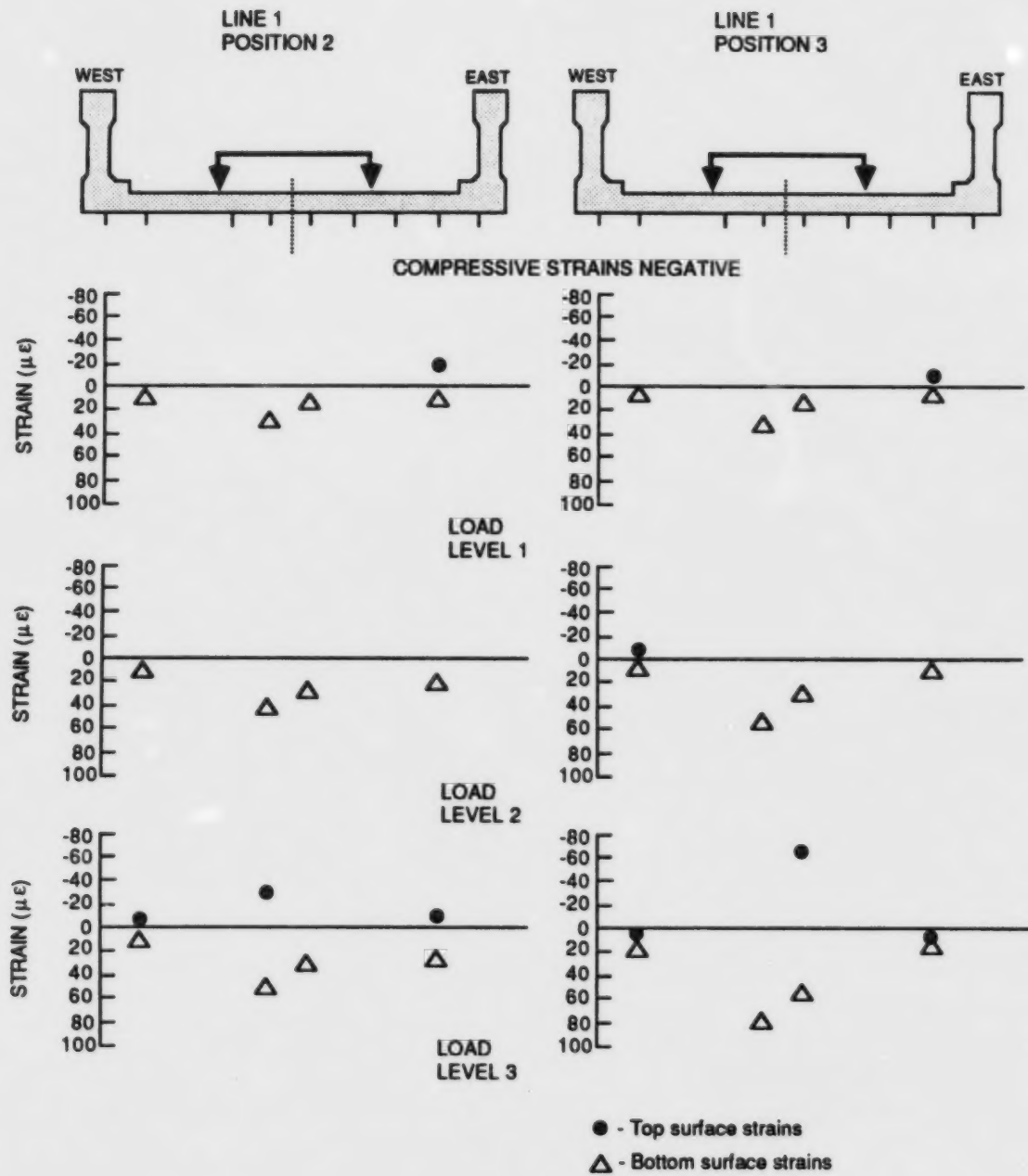


Figure 6(a)/ Transverse Strains at Section B, Symmetric Loading

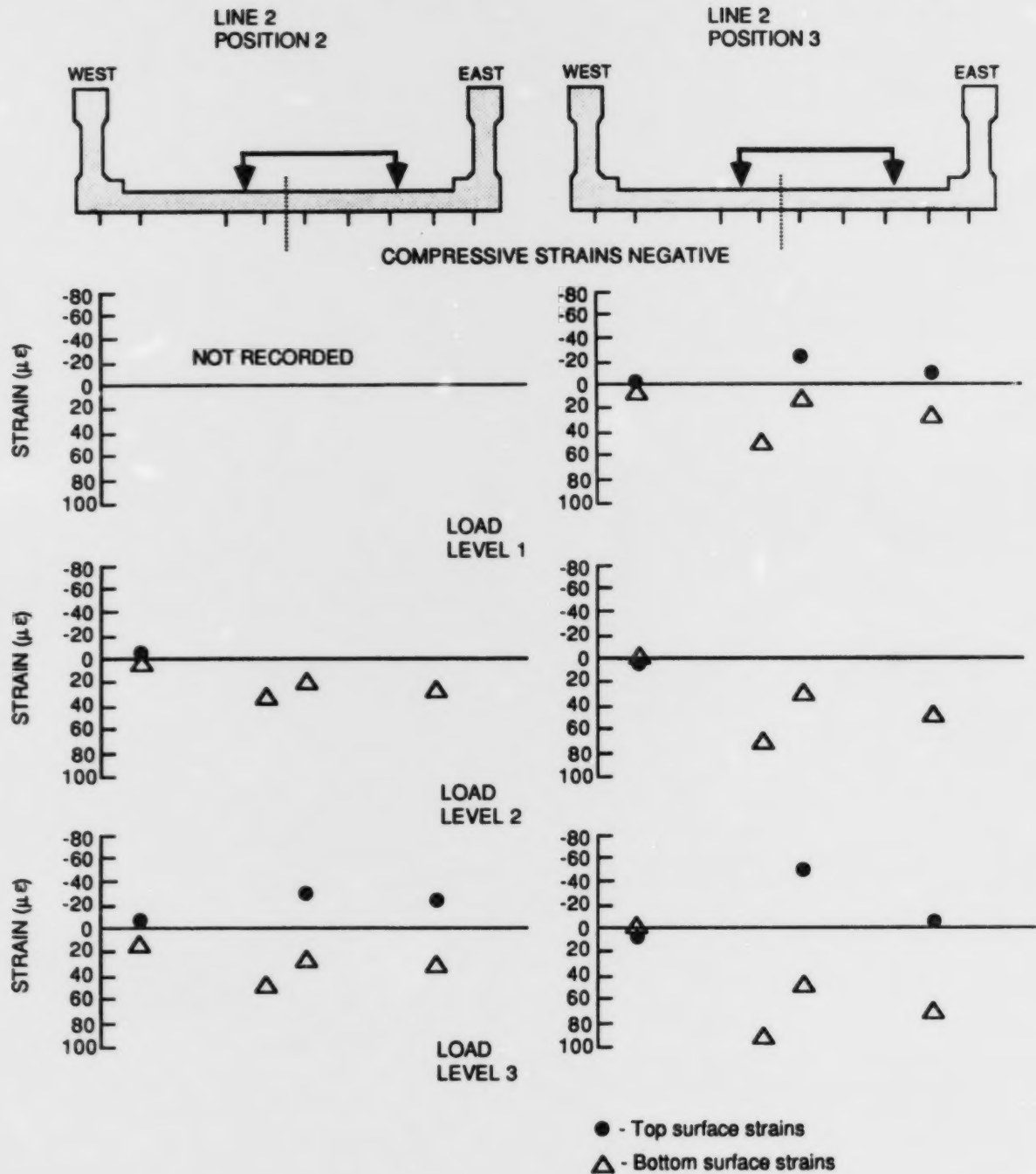


Figure 6(b)/ Transverse Strains at Section B, Eccentric Loading

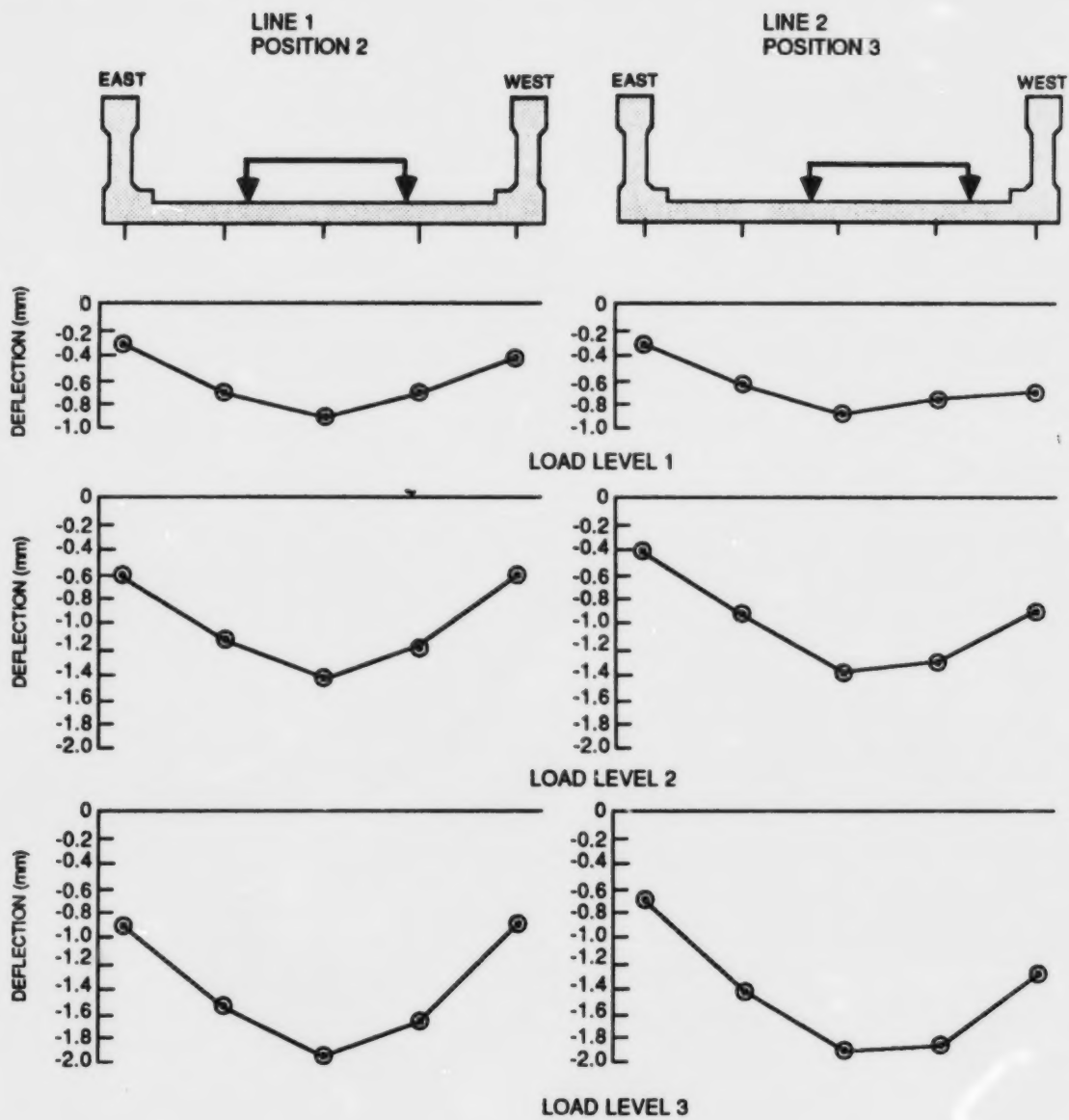


Figure 7/ Midspan Deflections